13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1816

BRIDGES IN KENTUCKY'S SEISMIC AREAS

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SUMMARY

The Commonwealth of Kentucky is located in a region which is greatly influenced by four seismic zones. Hundreds of bridges in areas influenced by the seismic zones were designed and constructed prior to stringent seismic codes. This paper reports on the development of site specific time histories, response spectra and seismic acceleration maps for the 120 counties in the state of Kentucky. The seismic input is used in the evaluation of bridges within Kentucky, and retrofits are recommended when required. Field testing, one-dimensional to three-dimensional computer modeling and analytical model calibration are also reported. The Tennessee River tied-arch bridge is selected to illustrate the methodology followed in bridge evaluation.

INTRODUCTION

Kentucky is influenced by the New Madrid Zone, Wabash Valley, Giles County (Virginia), and Eastern Tennessee Seismic Zones. In 1811- 1812, four of the most severe earthquakes in American history occurred in the New Madrid Seismic Zone. Recent observations and seismic measurements indicate that the New Madrid Seismic Zone is still the most hazardous zone in the east of the Rocky Mountains (Johnston [1]).

There are hundreds of bridges in the Commonwealth of Kentucky which were designed and constructed prior to the application of present-day seismic design codes. These existing older bridges were not designed to resist seismic loadings and have not yet been subjected to any moderate or strong earthquake. Seismic evaluation and retrofit of these bridges is currently being carried out. The following topics will be presented via case studies on bridges in Kentucky: 1) Development of site specific time histories, response spectra, and seismic acceleration maps for the 120 counties in Kentucky; 2) Field testing (ambient vibrations) of over ten long-span bridges; 3) One-dimensional to three-dimensional computer modeling of short and long span bridges; 4) Seismic evaluation of more than 500 bridges in Western Kentucky; and 5) Retrofit of bridges.

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SITE SPECIFIC TIME HISTORIES, RESPONSE SPECTRA AND SEISMIC ACCELERATION MAPS

Time histories of hypothetical earthquakes are derived based on the stochastic simulation model proposed by Boore [2], and recommended by Reiter [3]. Several factors have been taken into consideration such as the probability of earthquakes from nearby seismic zones, the attenuation of ground motions with distance in the Central United States, and the possibility of a random event occurring outside of the generally recognized zones of seismicity in the area. Using the random-vibration modeling, time histories and response spectra for the 50, 250, and 500-year earthquake events are determined for all of the county seats in Kentucky. Time histories representing the credible earthquake event are generated for the vertical and two orthogonal horizontal directions. The definition of the 250-year event is: the peak horizontal particle acceleration, at the top of bedrock which has a 90% probability of not being exceeded in 250 years (i.e. 10% probability of exceedance). A recurrence rate (return period) can be calculated for an earthquake, which would produce the 250-year event. The time histories and associated 0 and 5 percent damped response spectra are used to establish ground motion zones within Kentucky. Boundaries of the zones coincide with county lines and are intended to reflect differences in peak particle motion as well as the duration of the ground motions. The seismic acceleration map of the 250-year earthquake event for the 120 counties in the state of Kentucky is shown in Figure 1. For the seismic zones affecting Western Kentucky, the 50-year, and 500-year earthquake events defined by Street et. al. [4] correspond to the AASHTO design earthquake and near the maximum credible earthquake, respectively. The results obtained will be used as guidelines for preliminary seismic screening and detailed seismic evaluation, and the design of new bridges within Kentucky.

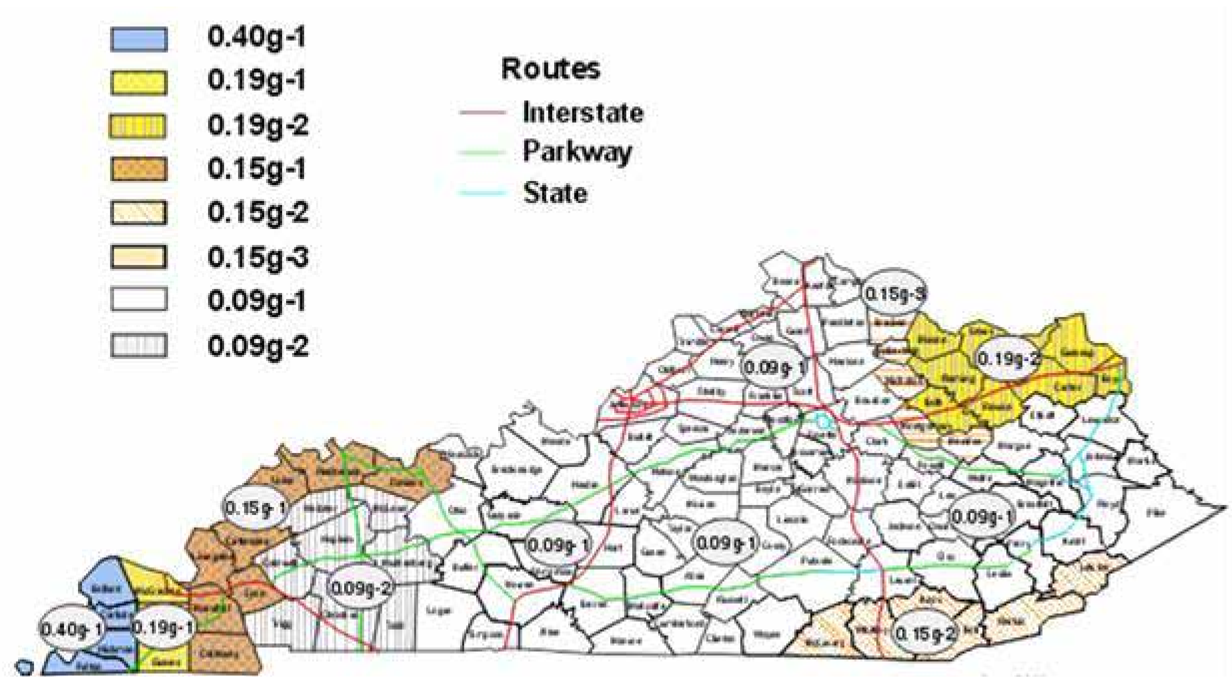


Figure 1 Seismic Acceleration Map of the 250-year Earthquake Event for the Commonwealth of Kentucky

FIELD AMBIENT VIBRATION TESTING

Assessment of a bridge is dependent on its as-built or current conditions and requires constructing a finite element (FE) model to assess the dynamic response of the bridge during a projected seismic event. The process generally includes a preliminary finite element modeling, followed by a non-destructive dynamic and/or static field testing, and calibration of the FE model. Some long-span bridges in Western Kentucky are influenced by the seismically active New Madrid Zone, and thus require field testing to better evaluate their dynamic behavior. To date, field testing has been carried out on more than ten long-span bridges. Examples of these bridges are the US-51 Bridge in Ballard County (Figure 2a), the US-41 Southbound and Northbound Bridges between Henderson, Kentucky and Evansville, Indiana; the Brent-Spence Bridge on Interstate 75 (Figure 2b); the Sherman-Minton Bridge on Interstate 64 (Figure 2c); the Cumberland River Bridge and the Tennessee River Bridges on Interstate 24. Field testing has been also carried out on several bridges in Kentucky in order to construct the finite element baseline models. Examples of such bridges are the Roebling suspension bridge on KY-17 over the Ohio River, the Maysville cable-stayed bridge connecting Maysville in Kentucky and Aberdeen in Ohio over the Ohio River (Figure 2d), and the Owensboro cable-stayed bridge connecting Owensboro in Kentucky and Rockport in Indiana. A representative example of field testing of bridges in Kentucky will be presented for the Tennessee River Bridge.

(a) The US-51 Bridge

(b) The Brent-Spence Bridge

(c) The Sherman-Minton Bridge

(d) The Maysville Bridge

Figure 2 Long-span Bridges in Kentucky

The Tennessee River Bridges shown in Figure 3, located on Interstate 24 in Western Kentucky, are steel plate-girder tied-arch bridges. Each entire bridge consists of nine spans symmetrically located on both sides of the arch span with the total length of 643 meters. The main span of the bridge is a steel tied-arch with a length of 163 meters. The superstructure of the bridge consists of the vertical load system, the lateral load system, and the floor system. The bracing system is a combination of transverse and diagonal bracings. Two wall type piers and arch configuration support the main span of bridge. The 26 main suspended steel wire ropes are vertically attached on both sides of the arch and floor system. The floor system consists of a 203.2 mm thickness concrete slab supported by five longitudinal stringers. The stringers are placed on the transverse built-up floor beams and braced by four transverse members. For the main arch span, the superstructure is supported by expansion bearings and fixed bearings. The expansion bearings permit translation and rotation whereas the fixed bearings allow only rotation.

(a) View of the bridges (b) Side view of a tied-arch main span

Figure 3 The Tennessee River Bridges

The field modal testing of a main arch span of the Tennessee River Bridges is carried out using the method of ambient vibration. The equipment used to measure the acceleration-time responses of instrumentation consists of tri-axial accelerometers linked to its own data acquisition system. The system contains a Keithly MetraByte 1800HC digital recording strong motion accelerograph. Two units contain internal accelerometers, while the two remaining units are connected to Columbia Research Labs, SA-107 force balance accelerometers. Sets of three accelerometers are mounted to aluminum blocks in orthogonal directions. A block is positioned at each station with the accelerometers oriented in the vertical, transverse and longitudinal directions. Accelerometers are connected to the data acquisition system by shielded cables. All measurements are taken by placing the instruments on the pavement due to the limited access to the actual floor beams and the time constraints involved. Measurement stations are chosen to both ends of the arch span and each joint of suspenders connected to the bridge deck. As a result, a total of 30 locations (15 points per side) are measured. Eight test setups are conceived to cover the planned testing area of the arch span of the bridge. A reference location, hereinafter referred as the base station, is selected based on the mode shapes from the preliminary finite element model. Each setup consists of three base triaxial accelerometer stations and four moveable triaxial accelerometer stations. The sampling frequency on site is chosen to as high as 1,000 Hz to capture the short-time (higher-frequency) transient signals of the ambient vibration in detail. The ambient vibration measurement is simultaneously recorded for 60 seconds at all accelerometers, which resulted in total 60,000 data points per data set (channel). During all tests, normal traffic flow is permitted.

The data processing and modal identification of the tested steel arch bridge are carried out by MACEC, a modal analysis program for civil engineering construction (De Roeck [5]). The measured data are first detribalized which enables the removal of the DC-components that can badly influence the identification results. Then a re-sampling of the raw measurement data is necessary. It is important to proceed with this now, because afterwards other preprocessing steps will go much faster due to the reduced amount of data. A re-sampling and filter from 1000Hz to 25Hz is the same as decimating (=low-pass filtering and re-sampling at a lower rate) 40 times. The decimating 40 times of raw data results in 1,500 data points and an excellent frequency range from 0 till 12.5 Hz. A much nice power spectral density diagram can be obtained. A smaller interval would reduce the number of points too much. Then the data are ready for the system identification to extract the eigen-frequencies and mode shapes (Ren [6]).

Though the peak picking (PP) method is faster and provides a good identified frequency in most of cases, it sometimes cannot reflect enough good mode shapes. The stochastic subspace identification (SSI) in time domain is applied to the re-sampled data. One of the advantages of the SSI method is that the stabilization diagram can be constructed in an effective way. Afterwards models of increasing order are obtained by rejecting less singular values. The stabilization diagrams aid the engineer to select the true modes. The identified frequencies of an arch span of the Tennessee River Bridge are shown in Table 1.

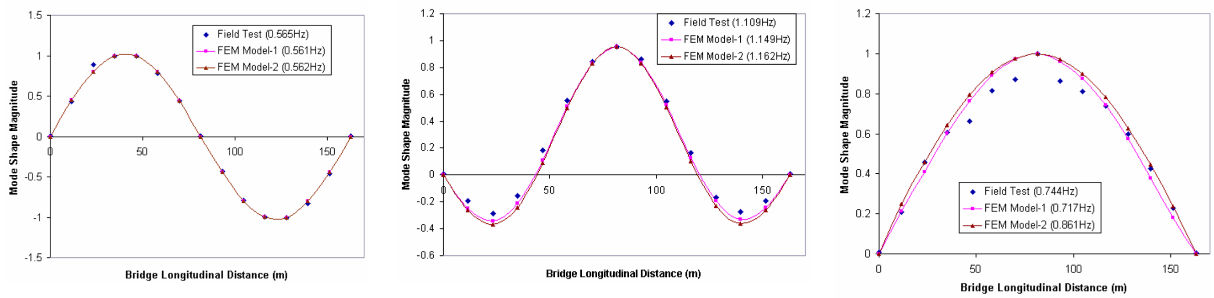
Table 1 Identified and Calculated Frequencies

Mode	Model-1 (Hz)	Model-2 (Hz)	Peak-Picking (Hz)	Stochastic Subspace Identification (Hz)
1st vertical	0.561	0.562	0.567	0.565
2nd vertical	1.149	1.162	1.100	1.109
3rd vertical	1.749	1.762	1.483	1.488
1st transverse	0.717	0.861	0.767	0.744
2nd transverse	1.557	2.897	1.267	1.242
3rd transverse	1.838	3.242	2.300	2.301
1st longitudinal	1.516	1.573	1.583	1.563

COMPUTER MODELING AND CALIBRATION

Three-dimensional linear elastic finite element models of the arch span of the Tennessee River Bridge have been constructed using SAP2000 (Wilson [7]). The FE model is developed for both the analytical modal analysis and seismic response analysis. The arch members, girders, stringers, floor beams and bracing members are modeled by two-node frame elements that have three translational degrees of freedom (DOFs) and three rotational DOFs at each node. All suspended wire ropes are modeled by the truss element, a common frame element with released three rotational DOFs at each node. Wall type piers at two sides and at the top of cap are modeled as frame elements while the web walls are modeled as shell elements. Bridge bearings are modeled by a set of rigid elements connected the superstructure and piers to simulate the actual behavior. In order to study the effect of concrete slab in bridge deck system on finite element modeling, two FE models (Model-1 and Model-2) are conceived. In FE Model-1, the concrete slab element is simulated as equivalently lumped joint masses for modal analysis. In Model-2, the concrete slab deck is modeled by shell elements. As a result, the Model-1 has a total of 500 frame elements and 120 shell elements with 507 nodes. The Model-2 results in a total of 507 nodes, 500 frame elements and 176 shell elements.

Table 1 summarizes the identified and FE calculated frequencies. It is found that the analytical modal analysis results agree well with the field test results. The tested first transverse frequency is between two computed values of FE models but closer to that of model-1 with the joint lumped masses. The model-2 gives comparable vertical and longitudinal frequencies as same as the model-1. Considering the concrete slab of the deck system in the FE Model-2, it is seen that it will mainly influence the transverse behavior of the bridge. For example, the first transverse frequency of Model-2 is greater than that of Model-1 by 20%. Figure 4 shows the comparison of the first two vertical mode shapes and the first transverse mode shape. It can be seen that the computed results of bridge model-1 with the concrete slab simplified by concentrated joint masses are in good agreement with field test results. This simplified model is suitable to the seismic analysis of the main arch bridge.



(a) 1st vertical mode (b) 2nd vertical mode (c) 1st transverse mode

Figure 4 Comparison of the Mode Shapes

SEISMIC EVALUATION AND RETROFIT OF BRIDGES

Due to the importance of the interstates and main routes in the Commonwealth of Kentucky, seismic evaluation on more than 500 bridges against the credible earthquakes in their associated seismic zones is being carried out. Based on structural vulnerability and seismic hazard, a convenient and practical method of seismic ranking for regular bridges is recommended in the “Seismic Retrofitting Manual for Highway Bridges” (Buckle [8]). The regular bridges are defined as those having less than seven spans, no abrupt or unusual changes in weight, stiffness, or geometry, and no large changes in these parameters from span to span or from support to support (AASHTO [9]). Any bridge not satisfying these requirements is to be identified as irregular bridge. The purpose of preliminary screening is to identify those bridges which are seismically deficient and those in the greatest need of retrofitting. These preliminary screening results will be used as a basis to selecting bridges for more detailed quantitative evaluation. In order to work efficiently, Seismic Inventory of Bridges, a user-oriented database program according to the seismic retrofitting manual, has been successfully developed with Microsoft Access. It assists the user by providing a lot of help information during preliminary seismic screening. Seismic ranking computation of the bridge can be easily performed after inputting all the necessary information such as general information of the bridge, site and superstructure, columns and piers, abutments and bearings.

Rigorous analysis procedures, such as time history analysis, are required for irregular bridges and those regular bridges with high seismic ranking. In recent years, some long-span bridges in the Commonwealth of Kentucky have already been evaluated under different credible earthquakes. Two double deck bridges over the Ohio River, the Brent-Spence Bridge on Interstate 75, and the Sherman-Minton Bridge on Interstate 64, were evaluated for seismic events following the Loma Prieta earthquake and the collapse of portions of the double-deck segments of Interstate 80 and the Bay Bridge in San Francisco (Harik [10]). The study on the Brent-Spence Bridge reveals that the approach spans are vulnerable to loss of span failure during the maximum credible earthquake while the main bridge will resist the maximum credible earthquake in the elastic range without any damage or loss of span. Other examples of bridge seismic evaluation are the Ohio River bridge on US-51 at Wickliffe (Harik [11]), the US-41 Southbound and Northbound bridges at Henderson (Harik [12]), the Cumberland River Bridge and the Tennessee River Bridges on Interstate 24.

Since the Tennessee River Bridges carry Interstate 24, the bridge is to be evaluated for the credible 250-year event and the maximum credible 500-year event earthquakes. During a 250-year event, the bridge is required to remain in the elastic range without any disruption to traffic. During a 500-year event, partial damages of the bridge are permitted. However, the bridge has to provide access to emergency services. In order to achieve the objective, seismic evaluation on the main bridge and approach spans is conducted by using time-history analysis and response-spectrum method, respectively.

In consideration of the bridge location, the border of Marshall and Livingston Counties in Kentucky, a time-history with peak horizontal acceleration of 15% gravity is considered. The time-history for the maximum credible earthquake (500-year event) has a peak horizontal acceleration of 30% gravity in Marshall and Livingston Counties. The earthquake duration is 20.5 seconds consisting of 4,100 data points at 0.005-second intervals. For a 250-year event, the peak ground accelerations along longitudinal, transverse and vertical directions are 165, 147 and 164 cm/sec^2, respectively. For a 500-year event, the peak ground accelerations along longitudinal, transverse and vertical directions are 289.19, 178.36 and 290.59 cm/sec^2, respectively. In seismic analysis, the bridge structure is subjected to all the three orthogonal (longitudinal, transverse and vertical) components for each event at 5% damping simultaneously. Two combinations of the components for each event are considered. The longitudinal and transverse components of the event are placed along two main directions (x–direction and y–direction) of the bridge, respectively. In above two combinations, vertical component is applied along the vertical direction (z–direction) of the bridge.

For the main bridge, seismic response analyses have been conducted using time-history analysis method. In all cases, stresses for selected critical arch members are well below the yield stress of the structural steel for different seismic excitation combinations. The maximum shear stress of the substructure at the bottom of the piers, resulting only from earthquake loads, is found to be very small. For the bearing supports over the piers, the seismic shear force Capacity/Demand (C/D) ratios are greater than 1.0 in the case of 250-year event. But these C/D ratios are less than 1.0 under the maximum 500-year earthquake event. Therefore, the supports with fixed bearings on the pier of the main bridge need to be retrofitted under maximum 500-year event. All the displacement C/D ratios are greater than 1.0 and hence the loss of span at the supports cannot occur.

Seismic analyses of the simplified models for the approach spans are carried out using the response spectrum method. The displacement C/D ratios are greater than 1.0 and loss of span at the supports of the approach bridge cannot occur. However, computed results show that the seismic shear force $r_{C/D}$ ratios of the anchor bolts for almost all of the bearings during the credible earthquakes are less than 1.0 and hence potential shear failure of these bolts exists. Therefore, the shear capacity of these bearings can be increased by providing additional anchor bolts or by replacing existing anchor bolts, or by replacing the bearings with seismic isolation bearings. For the bridge bearings with $r_{C/D}$ <0.5, retrofitting is strongly recommended.

CONCLUSIONS

Seismic evaluation of more than 500 bridges in Kentucky, USA, has been and is being carried out. Field ambient vibration testing of bridges has been conducted for over ten long-span bridges in recent years, which enables the researchers and engineers to obtain accurate as-built dynamic characteristics and calibrate the preliminary finite element models. The calibrated models can then be used as the baseline models to perform the further seismic analysis and evaluation of bridges. A case study of the Tennessee River Bridge is presented. Analytical results indicate that the main bridge will survive the credible 250-year and 500-year earthquakes without significant damage and no loss of span. But the supports with fixed bearings on the pier of the main bridge need to be retrofitted under 500-year earthquake event.

Some supports on the approach spans are found to be vulnerable to shear failure of anchor bolts under credible 250-year earthquake. All supports on the piers of the approach spans need to be retrofitted under 500-year earthquake event. Additional anchor bolts or other retrofit measures at the bearings are recommended.

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